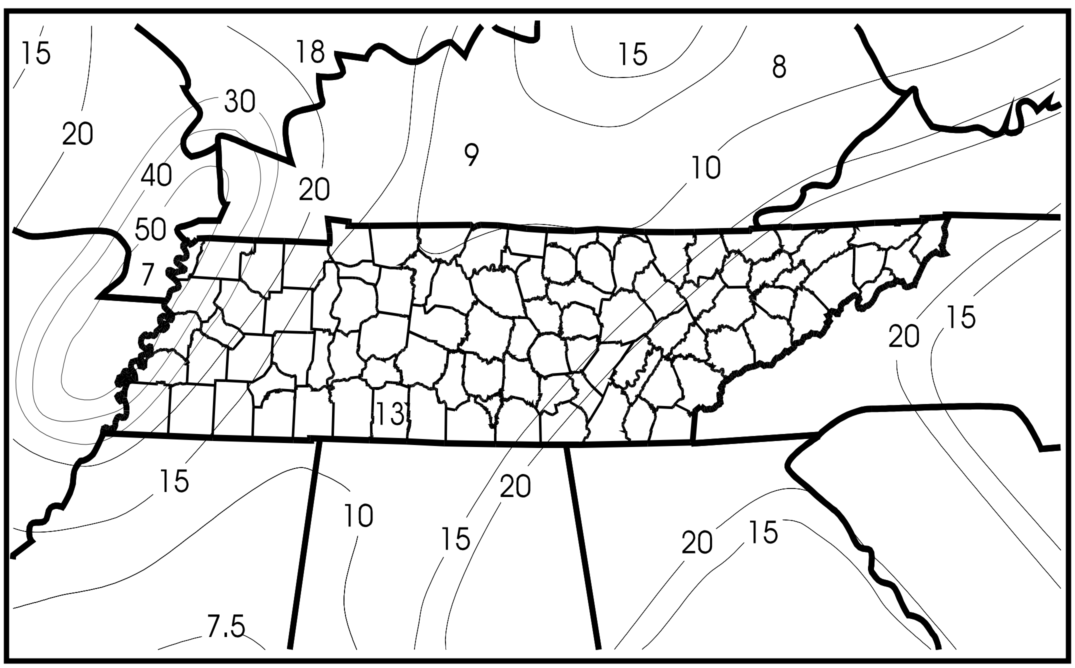
Tennessee Division of Solid Waste Management:

Earthquake Evaluation

Guidance Document



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FOR

EARTHQUAKE EVALUATION GUIDANCE POLICY

EARTHQUAKE EVALUATION GUIDANCE POLICY

INTRODUCTION

The promulgation and approval by the Environmental Protection Agency of the State of Tennessee Solid Waste regulations has added an earthquake evaluation requirement for Class I and Class II solid waste landfills located in Tennessee. Specifically, the regulations state that "new Class I and Class II Solid Waste landfill facilities shall not be located in **seismic impact zones**, unless the owner or operator demonstrates that all containment structures, including liners, leachate collection systems, and surface water control systems, are designed to resist the **maximum horizontal acceleration** in **lithified earth material** for the site. The owner or operator must place the demonstration on the Narrative Description of the Facility and Operations Manual." In order to comply with this regulation it is first necessary to understand the meaning of the terms " maximum horizontal acceleration", "lithified earth material" and "seismic impact zone", which are defined in the regulations as follows;

"maximum horizontal acceleration in lithified earth material" means the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment.

"lithified earth materials" means all rock, including all naturally occurring and naturally formed aggregates of masses of minerals or small particles of older rock that formed by crystallization of magma or by induration of loose sediments. This term does not include man-made materials, such as fill, concrete, and asphalt, or UNCONSOLIDATED earth materials, soil, or regolith lying at or near the earth surface.

"seismic impact zone" means an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth materials, expressed as a fraction of the earth's gravitational pull will exceed 0.10g in 250 years.

In order to implement the above stated regulation it is first necessary to determine the **maximum horizontal acceleration in the lithified earth materials** at a proposed or at an existing site so as to determine if the site is in fact within a "seismic impact zone".

DETERMINATION OF HORIZONTAL GROUND ACCELERATION

There are a number of seismic hazard maps that have been developed to depict horizontal accelerations by means of contour lines. The United States Geological Survey has developed a generalized map of the United States that depicts the expected horizontal ground accelerations (with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years) for the entire United States referred to as Open-File No. 82-1033 (Map 1). The regulations also state that the maximum horizontal acceleration may also be determined by performing s site-specific seismic risk assessment which is based upon a probablistic approach.

EVALUATION OF SEISMIC FAILURE MECHANISMS

Basically, there are two potential mechanisms by which solid waste landfill facilities may fail as a result of earthquake induced ground motions. These two potential failure mechanisms are referred to as liquefaction and slope stability. Although this guidance policy will present one procedure for evaluating liquefaction and one for evaluating global slope stability, there is actually more than one procedure for evaluating the potential of each of these phenomenon.

EVALUATION OF EARTHQUAKE FORCES ON THE SLOPE STABILITY OF SOLID WASTE LANDFILLS

A review by the Tennessee Department of Solid Waste Management of the current literature on the slope stability of landfill slopes has resulted in the adoption of a basic procedure for the evaluation of global and veneer stability of landfill slopes and cover systems in areas of moderate (.15g) to high (above .2g) seismic accelerations. The adopted procedure for determining global stability of waste slopes was developed by Newmark and later refined by Makdisi and Seed to determine the amount of deformation an earthen embankment may undergo as a result of seismic forces. Although a procedure for determining veneer stability of the cover system has been adopted, global type failures are the main focus of this guidance policy since these type failures generally are a catastrophic nature.

The **revised NEWMARK PROCEDURE** for slope stability analysis of waste slopes is as follows:

STEP 1. Develop a model of the landfill slope configurations to be used for pseudo-static analysis.

STEP 2. Determine the maximum undrained shear strengths of the soil and waste layers within the landfill mode.

STEP 3. Multiply the maximum undrained shear strengths of the soil and waste layers within the landfill model.

STEP 4. Perform pseudo-static analyses on the landfill model substituting different values for the horizontal acceleration so as to determine which acceleration results in a factor of safety of one. The horizontal acceleration that yields a factor of safety of one shall be referred to as the yield acceleration (k_y). It should be noted that the Tennessee Division of Solid Waste Management utilizes STABL5M to evaluate the stability of landfill slopes.

NEWMARK PROCEDURE (CONTINUED)

STEP 5. Determine the maximum of crest acceleration (u_{max}) induced in the embankment and the natural period (To) of the embankment. This can be accomplished by several different methods which include the following:

I. a finite element analysis of the embankment section (Clough and Chopra, 1966; Idress and Seed, 1967)

II. by a shear slice analysis (Ambraseys, 1960; Seed and Martin, 1966).

III. a simplified approach developed by Makdisi and Seed that lends itself to hand calculations is presented in the following paragraphs.

(Makdisi / Seed Simplified Procedure to Determine Crest Acceleration and Period)

STEP 5a. Determine the following embankment and subsurface soil properties;

Height of embankment, h (ft)

Unit weight of waste fill materials, γ_g (pcf) and site soils, γ_{soil}

Mass density, $\rho = \gamma$ / 32.2 ft./sec

Maximum shear wave velocity, v_{max} (obtain from crosshole velocity survey or from approximations using the following relationships):

G_{max} = 65 x N value of the site soils (Note: G_{max} is in TSF)

$G_{max} / \rho)^{1/2} - V_{max}$

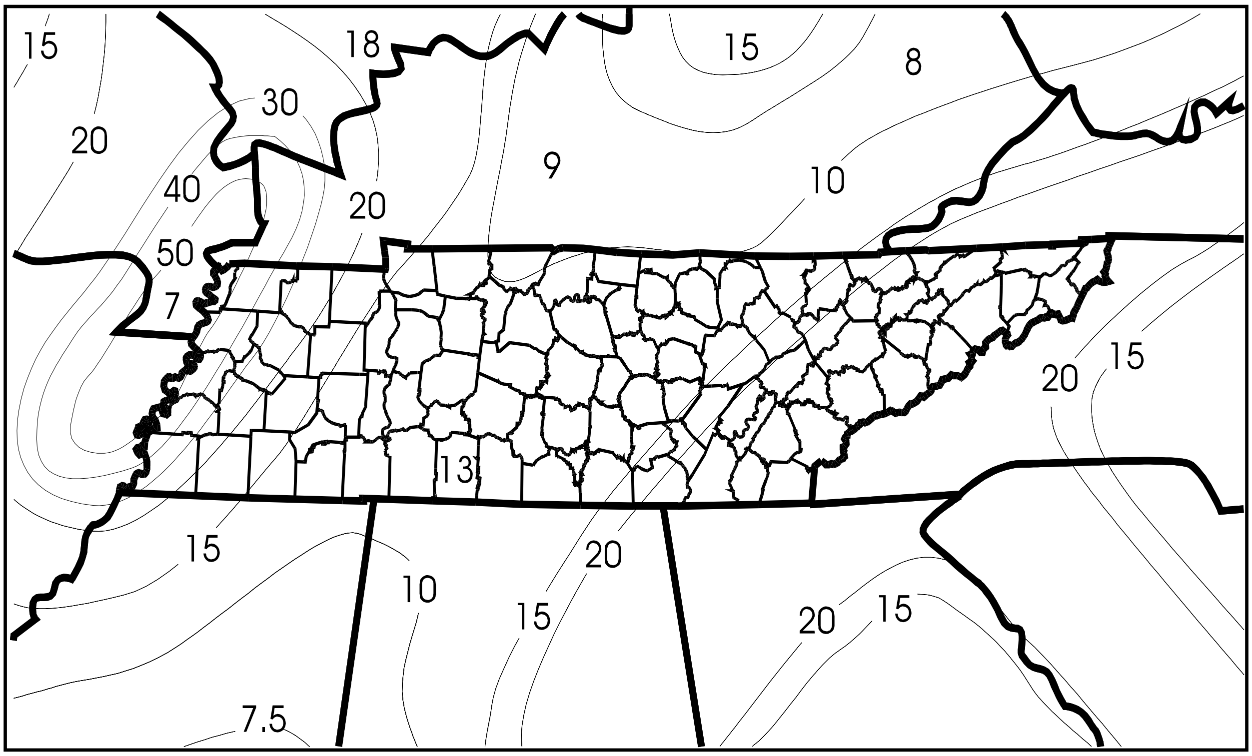
Maximum Horizontal Acceleration, a_{max} (obtain from USGS map)

STEP 5b. Perform First Iteration

I. Assume value of v_s

II. Calculate $G/G_{max} = (VS/V_{max})^2$

III. From Figure I determine the shear strain (ε) and damping, (λ)



USGS MAP OF MAXIMUM HORIZONTAL ACCELERATIONS

(90 PERCENT PROBABILITY OF NOT BEING EXCEEDED IN 250 YEARS)

NOTE: ACCELERATIONS ARE EXPRESSED IN PERCENT OF GRAVITY

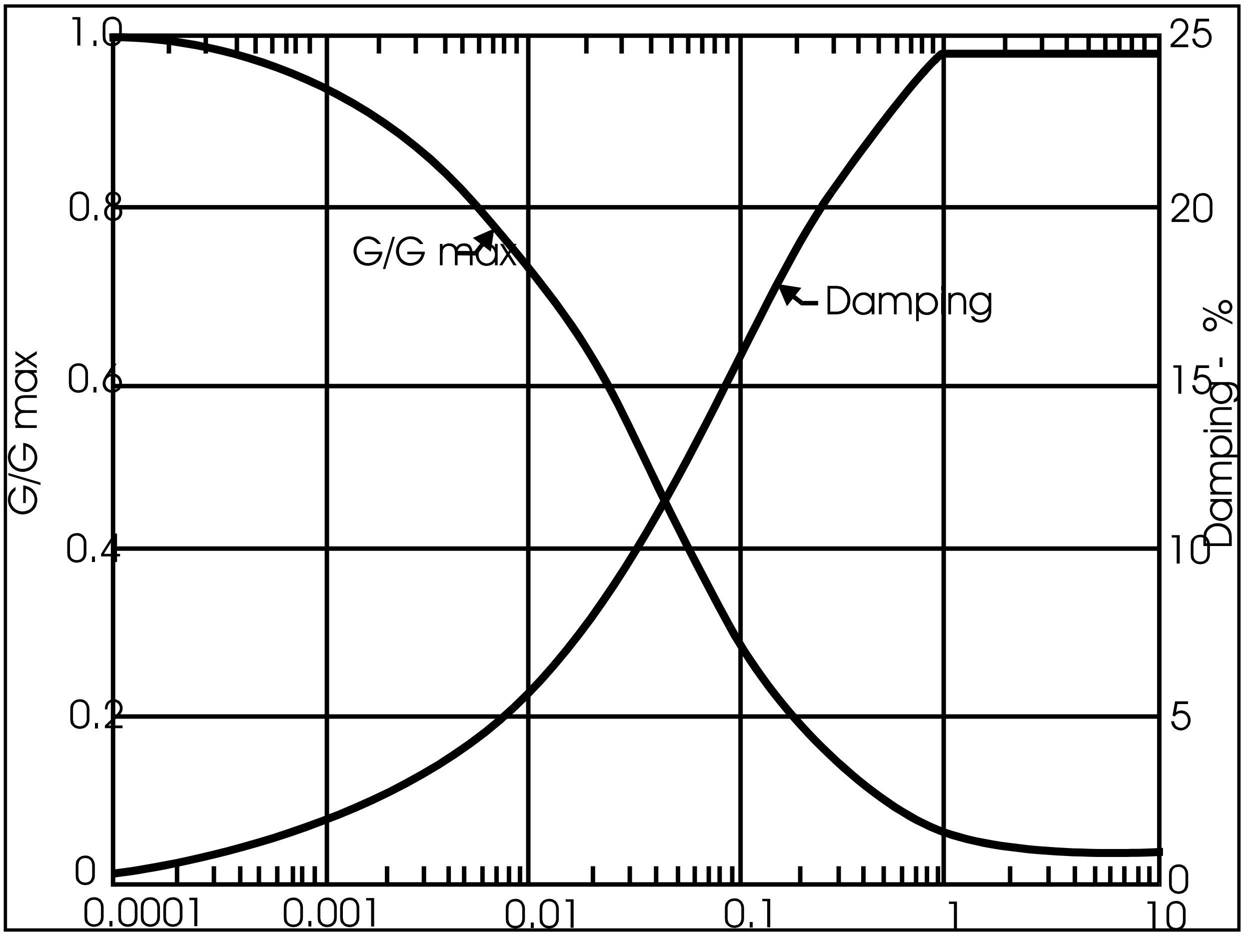


FIGURE 1: SHEAR MODULUS AND DAMPING CHARACTERISTICS USED IN RESPONSE CALCULATIONS

STEP 5B. First Iteration for determining crest acceleration (continued)

IV. Calculated the values of the first natural frequencies (ω) and the associated natural periods (T) as follows;

$\omega_1 = 2.4\ (V_s / h) =$ rad/sec, $T_1 = 2\pi / \omega_1$

$\omega_2 = 5.52\ (V_s / h) =$ rad/sec, $T_2 = 2\pi / \omega_2$

$\omega_3 = 8.65\ (V_s / h) =$ rad/sec, $T_3 = 2\pi / \omega_3$

V. Determine the spectral acceleration for the first three frequencies.

Using the periods determined in step IV, the percent damping from Figure 1, and the maximum horizontal acceleration from the USGS Map (MF 2120), find the corresponding spectral accelerations (S_{an}) from Figure 2.

VI. Calculate the maximum crest accelerations (u_{max}) for the first three modes: (NOTE: ϕ is referred to as a mode participation factor)

GIVEN:

$\phi_1 = 1.6$, $\phi_2 = 1.06$, $\phi_2 = 0.86$

FIND:

$u_{1\,max} = {}^{\phi}1\ (S_{a1})$

$u_{2\,max} = {}^{\phi}2\ (S_{a2})$

$u_{3\,max} = {}^{\phi}3\ (S_{a3})$

VII. Determine the maximum value of the crest acceleration by taking the square root of the sum of the squares of the maximum accelerations of the first three modes.

$[(u_{1\,max})^2 + (u_{2\,max})^2 + (u_{3\,max})^2]^{1/2} = u_{max}$

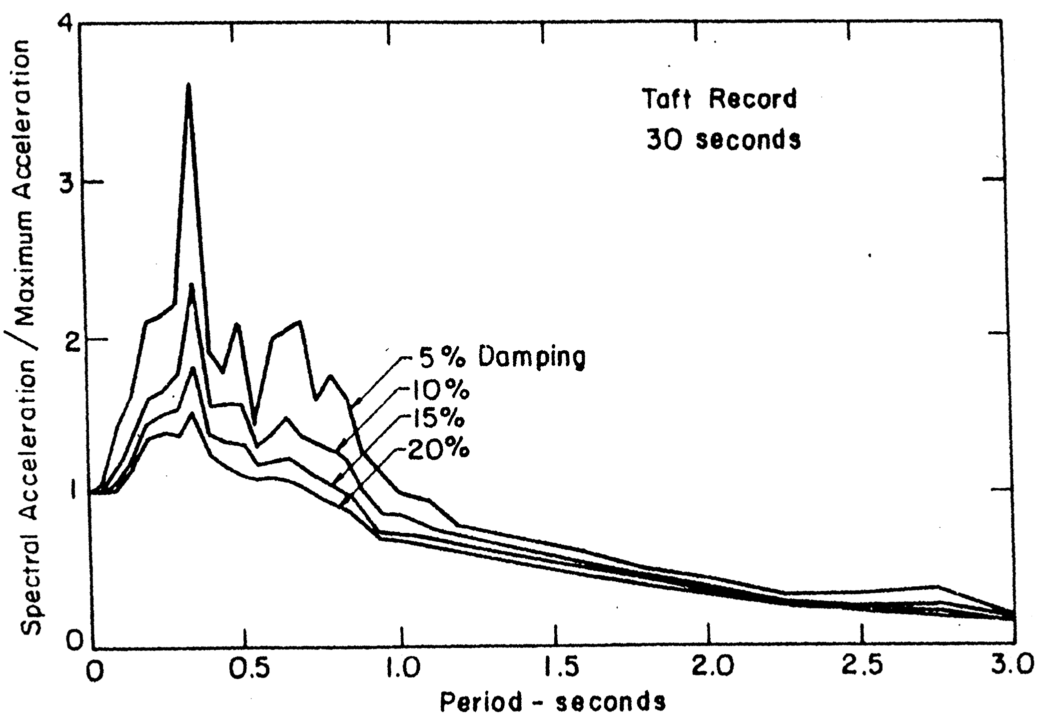


FIGURE 2: NORMALIZED ACCELERATION RESPONSE SPECTRA – TAFT RECORD (NORTH – SOUTH COMPONENT)

STEP 5B First Iteration for determining crest acceleration (continued)

VIII. Calculated the average equivalent shear strain $(\gamma_{ave})_{eq}$ from the following equation;

$(\gamma_{ave})_{eq} = 0.65 \times 0.3 \times (h / V_s 2)\ (S_{a1})$

NOTE: The shear strain obtained from the above calculation is generally different from the shear strain determined from using assumed velocity values and entering Figure 1 as was done in step III of 5b. If there is a difference between the assumed shear strain values and the calculated values, it will be necessary to perform a new iteration using the value obtained from the above equation to determine a new set of Modulus and damping parameters. Generally, it will take three iterations for the strain compatible properties to converge.

STEP 5c. Perform a second iteration so as to determine crest acceleration

1. Enter Figure 4 with the shear strain determined in Step VIII of the first iteration and find G/G_{max} and the damping, (λ)
2. Determine V_s from the $G/G_{max} = (V_S/V_{max})^2$ relationship.
3. Repeat steps IV through VIII as in the first iteration.

STEP 5d. Perform a third iteration as described in sections 5a and 5c to obtain the maximum crest acceleration.

Upon determining the maximum value of the crest acceleration u_{max} proceed with the Newmark Procedure so as to calculate the total deformation at the site.

STEP 6. Determine the maximum of crest acceleration (k_{max}) for any level within the embankment using the maximum crest acceleration (u_{max}) determined in Step 5 and entering Figure 3.

[NOTE: THE NUMBER 0 IN THE y/h COLUMN IS THE CREST FOR THE EMBANKMENT.]

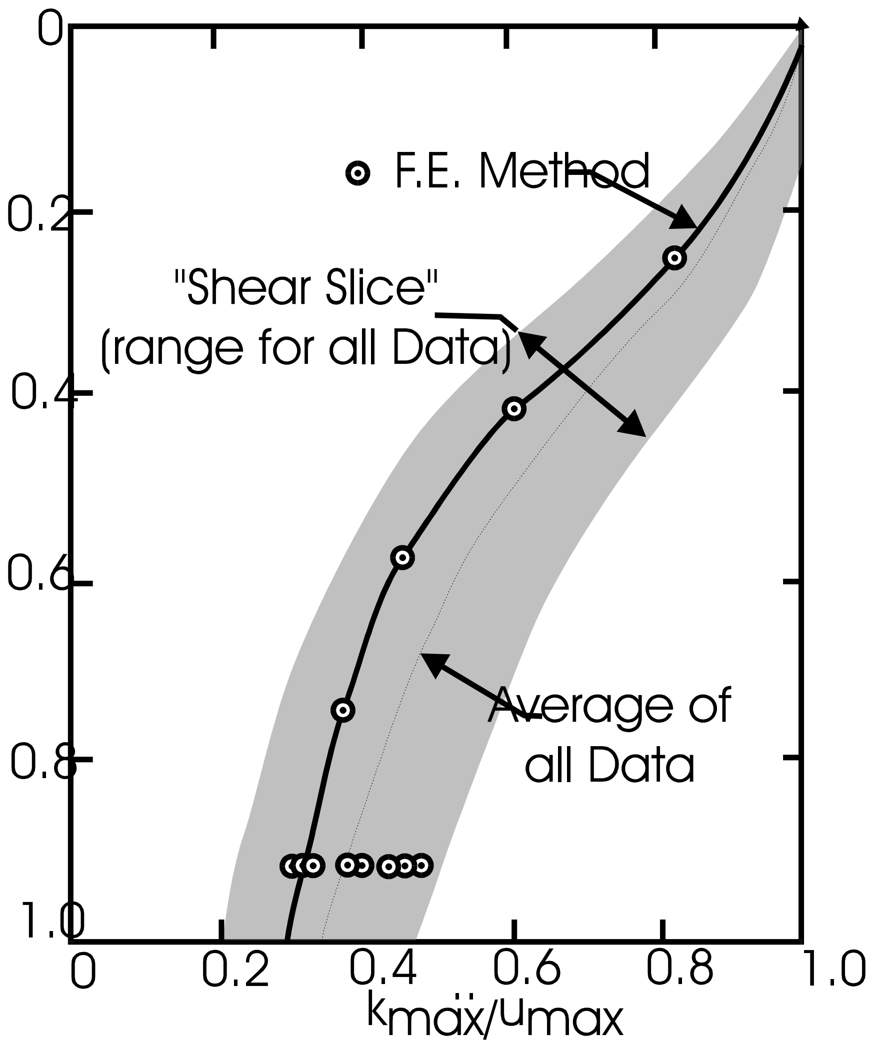


FIGURE 3: VARIATION OF A MAXIMUM ACCELERATION RATIO ≅ WITH DEPTH OF SLIDING MASS

STEP 7. Determine permanent displacement (U) for the yield acceleration (K_y) by entering Figure 4 with the appropriate values of k_{max} and T_o.

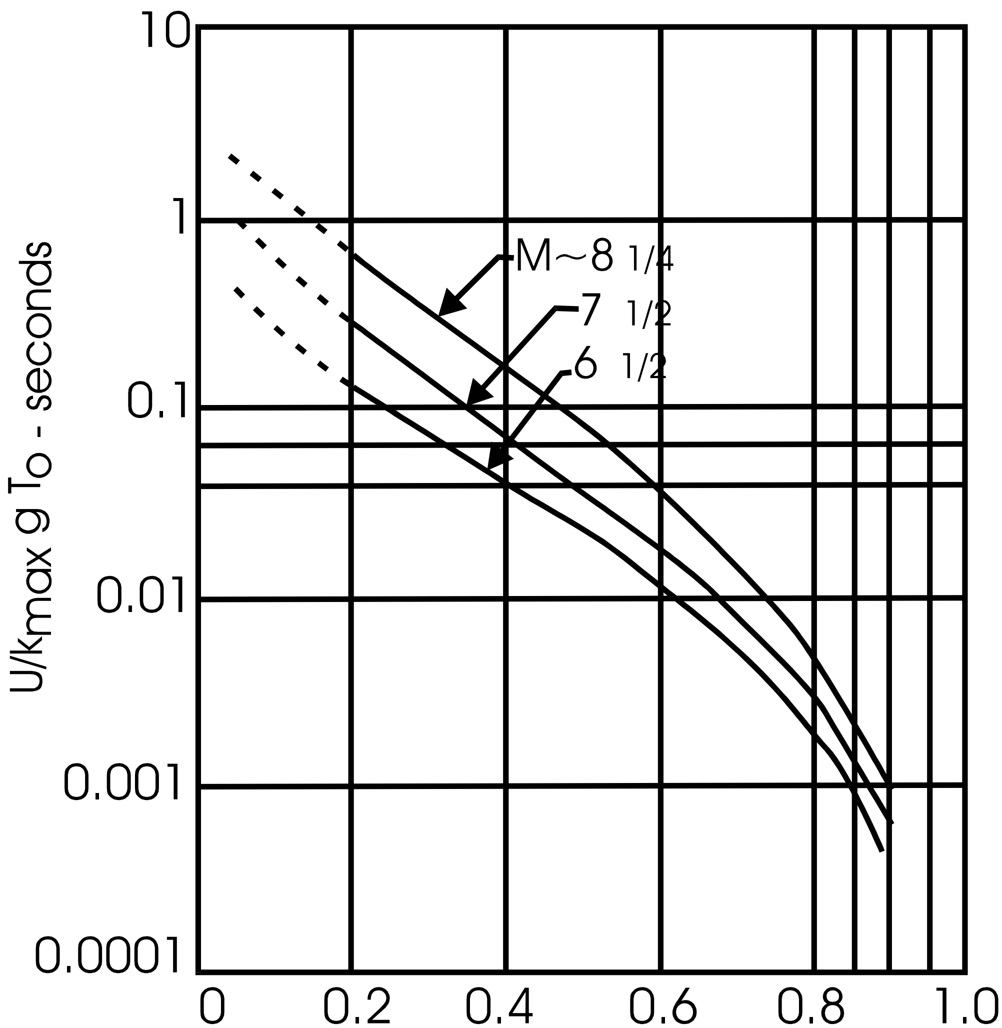


FIGURE 4: VARIATION OF AVERAGE NORMALIZED DISPLACEMENT WITH YIELD ACCELERATION

LIMITING SEISMIC SLOPE STABILITY DESIGN CRITERIA

The following limiting design criteria have been established so as to insure that the landfill liner, leachate collection system and landfill appurtenances will remain functional when subjected to earthquake induced forces.

1. Leachate collection systems and waste cells shall be designed to function without collection pipes for solid waste fill embankments that are predicted to undergo more than six inches of deformation.

2. No landfill shall be acceptable if the predicted seismic induced deformations within the waste fill exceed one-half the thickness of the clay liner component of the liner system.

VENEER STABILITY OF SOLID WASTE LANDFILL COVER SYSTEMS

In the event that geosynthetic type materials (geomembranes, geonets and geocomposites) are incorporated into cover systems at solid waste landfills it will be necessary to perform veneer stability type calculations. A quick basic check of the veneer stability of the cover system can be accomplished with the following equation:

$$\text{FACTOR OF SAFETY} = \frac{\text{RESISTING FORCES}}{\text{DRIVING FORCES}} = \frac{[\cos\alpha - a_{max}\sin\alpha]\tan\phi}{\sin\alpha + a_{max}\cos\alpha}$$

in which α is the slope angle, ϕ is the limiting interface friction angle and a_{max} is the pseudo-static seismic coefficient.

Again this type of stability analysis must result in a factor of safety that exceeds one to provide adequate stability against sliding. Presently, it is the opinion of the Solid Waste Division that his type of failure mechanism will generally not result in a catastrophic type of failure. Therefore, some flexibility will be given for the design of the stability of landfill cover systems.

APPENDIX I

GLOSSARY OF TERMS

ϕ is referred to as a mode participation factor

ρ - Mass density

γ - Unit weight

$(\gamma_{ave})_{eq}$ - average equivalent shear strain

(ε) - shear strain

(λ) - damping

(ω) - natural frequencies

(k_{max}) - maximum value of average acceleration

(u_{max}) - maximum crest acceleration

(S_{an}) - spectral accelerations

(T_o) - natural period

(U) - permanent displacements

a_{max} - Maximum Horizontal Acceleration

G - Shear Modulus of the soil

G_{max} - Maximum Shear Modulus for the soil

h (ft) - Height of embankment,

k_y - yield acceleration

N - standard penetration test value

v_{max} - Maximum shear wave velocity

v_s = shear wave velocity

EXAMPLE PROBLEM ONE (Determining Crest Acceleration and Permanent Deformation of a waste fill)

[Note: This example problem actually begins at Step 5 of the revised Newmark Method outlined in the preceding sections.]

GIVEN: k_y - yield acceleration = 0.12 g
h (ft) - Height of embankment = 150 ft.,
γ - Unit weight = 130 pcf
v_{max} (obtain from crosshole velocity survey or from approximations using the following relationship:

$G_{max} = 65$ N

$(G_{max} / \rho)^{1/2} = V_{max}$

a_{max} (obtain from USGS map) - Maximum Horizontal Acceleration = 0.2 g

FIND: (u_{max}) - maximum crest acceleration
(T_o) - natural period
(U) - permanent displacement of the waste fill

FIRST ITERATION:

STEP ONE: DETERMINE G/G_{max}, SHEAR STRAIN AND DAMPING

Assume $V_S = 600$ fps

and $G/G_{max} = (V_S/V_{max})^2 = 0.4$

From Figure 1: for $G/G_{max} = 0.4$ the shear strain = 0.06% and the damping (λ) = 13%

STEP TWO: CALCULATE THE NATURAL FREQUENCIES AND PERIOD

$\omega_1 = 2.4 (V_s / h) =$ rad/sec, $T_1 = 2\pi / \omega_1$

$\omega_1 = 2.4 (600 / 150) = 9.6$ rad/sec, $T_1 = 0.65$ sec

$\omega_2 = 5.52 (V_s / h) =$ rad/sec, $T_2 = 2\pi / \omega_2$

$\omega_2 = 5.52 (600 / 150) = 22.1$ rad/sec, $T_2 = 0.284$ sec

$\omega_3 = 8.65\ (V_s / h) =$ rad/sec, $T_1 = 2\pi / \omega_3$

$\omega_3 = 8.65\ (600 / 150) = 34.6$ rad/sec, $T_3 = 0.182$

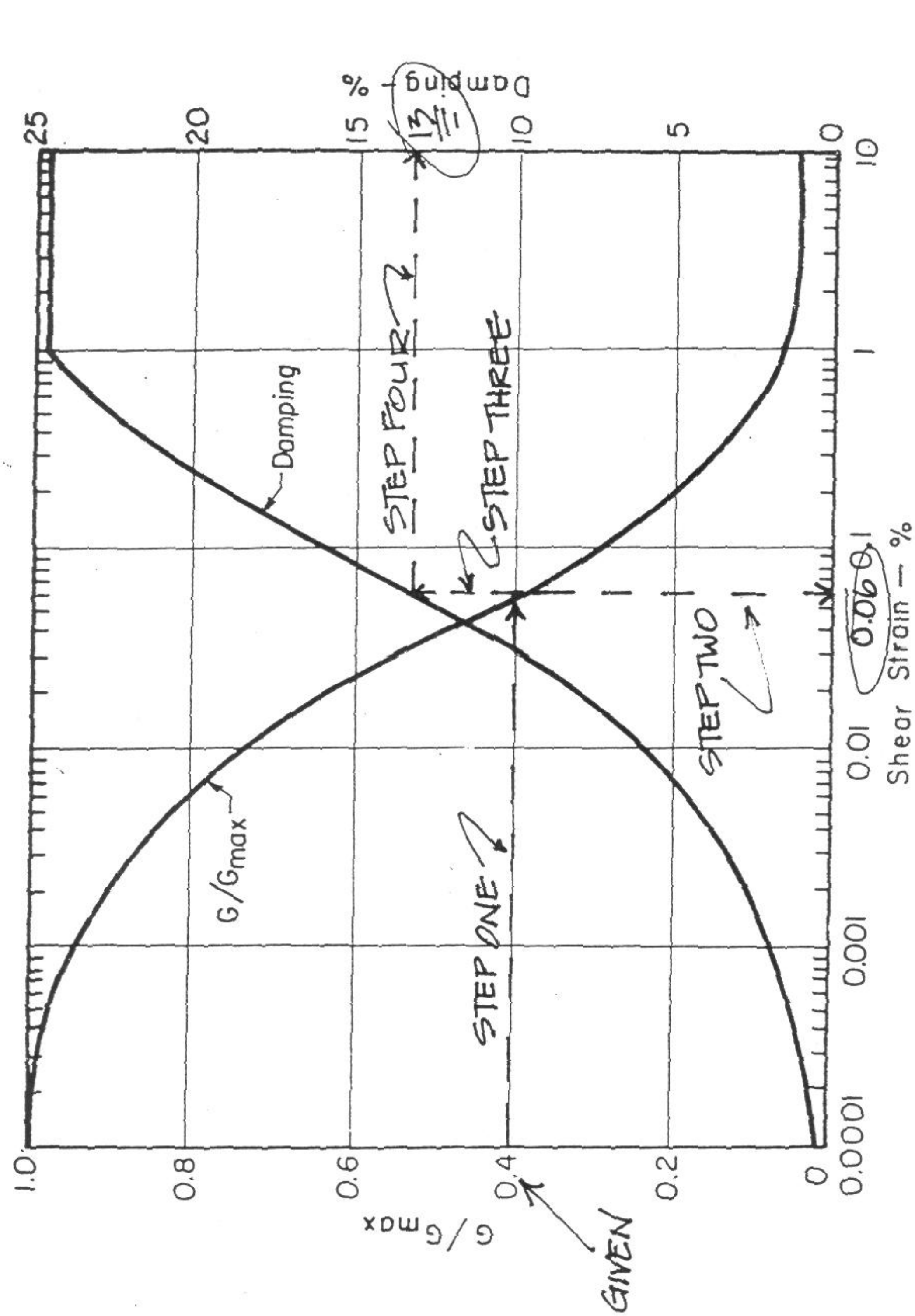


FIGURE 1: SHEAR MODULUS AND DAMPING CHARACTERISTICS USED IN RESPONSE CALCULATIONS

EXAMPLE PROBLEM ONE (CONTINUED)

STEP THREE: DETERMINE THE SPECTRA L ACCELERATIONS FOR THE THREE FREQUENCIES.

Use the value of the maximum horizontal acceleration (a_{max}) in combination with the value of λ determined in STEP ONE and the periods (T) determined in STEP TWO to enter Figure 2, to determine the spectral accelerations for each of the natural frequencies.

The actual values are as follows: $S_1 = 0.26$, $S_2 = 0.316$, $S_3 = 0.29$

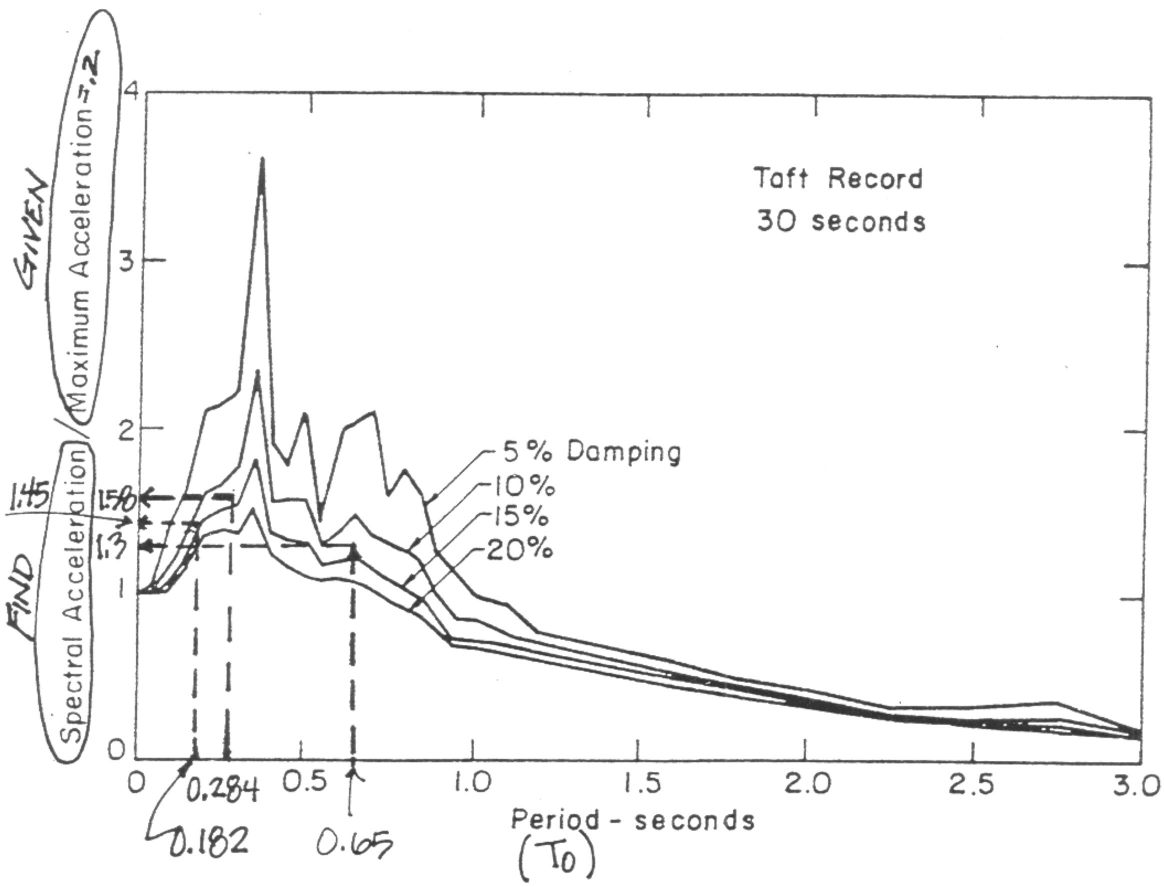


FIGURE 2: NORMALIZED ACCELERATION RESPONSE SPECTRA – TAFT RECORD (N – S COMPONENT)

EXAMPLE PROBLEM ONE (CONTINUED)

STEP FOUR: USE THE EQUATION PRESENTED IN STEP IV OF PART 5b TO DETERMINE THE MAXIMUM CREST ACCELERATIONS FOR EACH OF THE NATURAL FREQUENCIES.

$u_{1max} = 1.6 \times 0.26 = 0.416$ g

$u_{2max} = 1.06 \times 0.316 = 0.335$ g

$u_{3max} = 0.86 \times 0.290 = 0.249$ g

STEP FIVE: USE THE EQUATION PRESENTED IN STEP VII OF PART 5b TO DETERMINE THE MAXIMUM CREST ACCELERATION (u_{max}).

$$[(u_{1\,max})^2 + (u_{2\,max})^2 + (u_{3\,max})^2]^{1/2} = u_{max}$$

$$[(0.416)^2 + (0.335)^2 + (0.249)^2]^{1/2} = 0.59 \text{ g}$$

STEP SIX: CALCULATE THE AVERAGE EQUIVALENT SHEAR STRAIN $(\gamma_{ave})_{eq}$ FROM THE FOLLOWING EQUATION

$$(\gamma_{ave})_{eq} = 0.65 \text{ X } 0.3 \text{ X } h / V_S 2 \, (S_{a1})$$

$$(\gamma_{ave})_{eq} = 0.65 = 0.3 \times 150' / (600)^2 \, (0.26) \, (32.2) = 0.068\%$$

Note: Since the shear strain calculated from the above equation does not match the value determined in Step One it is necessary to perform a second iteration.

STEP SEVEN: PERFORM SECOND ITERATION

From Figure 1; for shear strain - .068% the $G/G_{max} = 0.36$
and the damping $(\lambda) = 13.7\%$

Thus $G/G_{max} = (V_S/V_{max})^2$ and so $V_S/V_{max} = 0.36^{1/2} = 0.6$ □ $V_S = 570$ fps

Therefore the frequencies are as follows:

$\omega_1 = 2.4 \, (570 / 150) = 9.12$ rad/sec, $T_1 = 0.69$ sec

$\omega_2 = 5.52 \, (570 / 150) = 20.97$ rad/sec, $T_2 = 0.3$ sec

$\omega_3 = 8.65 \, (570 / 150) = 32.87$ rad/sec, $T_3 = 0.19$ sec

EXAMPLE PROBLEM ONE (CONTINUED)

SECOND ITERATION (CONTINUED)

Determine the Spectral Accelerations

Use the value of the maximum horizontal acceleration (a_{max}) in combination with the value of (λ) determined in STEP SEVEN as well as the periods (T) determined in STEP SEVEN to enter Figure 2, to determine the spectral accelerations for each of the natural frequencies.

The spectral accelerations (S_{an}) from Figure 2 are as follows;

S_1 - 0.244, $S_2 = 0.32$, $S_3 = 0.294$

Determine the Crest Accelerations (u) for each of the natural frequencies (ω):

$u_{1max} = 1.6 \text{ x } 0.244 = 0.39 \text{ g}$

$u_{2max} = 1.06 \text{ x } 0.32 = 0.339 \text{ g}$

$u_{3max} = 0.86 \text{ x } 0.294 = 0.253 \text{ g}$

Therefore substituting into the following equation the maximum crest acceleration (u_{max}) is;

$[(0.39)^2 + (0.339)^2 + (0.253)^2]^{1/2} = 0.575\text{g}$

Finally substituting into the equation for maximum shear strain produces the following result;

$(\gamma_{ave})_{eq} = 0.65 \text{ x } 0.3 \text{ x } 150' / (570)2\ (0.244)\ (32.20 = 0.071\%$

STEP EIGHT: REPEATING THE SAME PROCEDURE FOR A THIRD ITERATION WILL RESULT IN THE FOLLOWING RESULTS:

$U_{max} = 0.57\text{g}$

$T_o = 0.7$ sec

$\gamma_{ave} = 0.07\%$

G = 1270 ksf

$\lambda = 14\%$

EXAMPLE PROBLEM ONE (CONTINUED)

STEP NINE: Determine the maximum value of average acceleration (k_{max}) for any level within the embankment using the maximum crest acceleration (u_{max}) determined in Step 8 and entering Figure 3. Since the height of the embankment is h = 150' y = (depth of failure plane) = 128' then y / h = .85. Upon entering Figure 3 at 0.95 and reading to the right yields a value for (k_{max}) / (u_{max}) of 0.35. Since u_{max} was found to equal .575g in the previous step then:

$$k_{max} = 0.35\ (.575g) = .201\ g$$

NOTE: y is the depth of the sliding and h is the height of the embankment

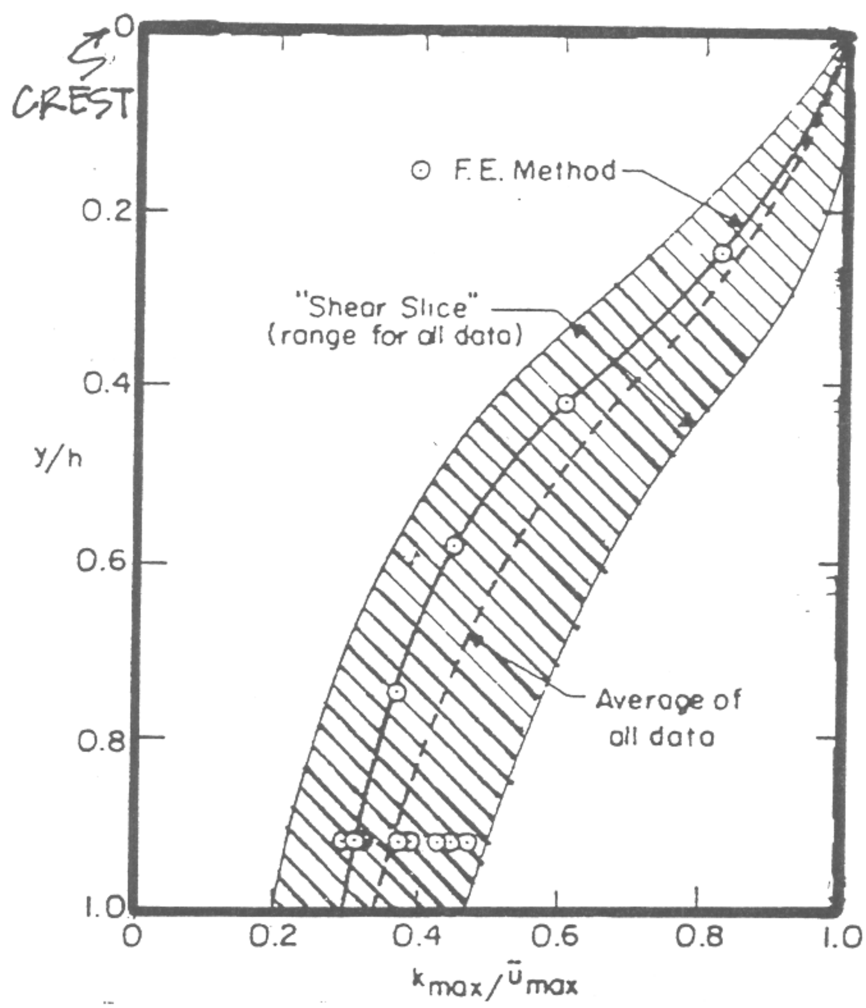


FIGURE 3: VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

EXAMPLE PROBLEM ONE (CONTINUED)

STEP NINE: Determine the permanent displacement (U) for the yield acceleration (K_y) by entering Figure 4 with the appropriate values of k_{max} and T_o.

CALCULATE: k_y / k_{max} = 0.12g / 0.201g = 0.597

FROM Figure 4: U / k_{max} (T_o) = 0.025

so that U = 0.025 [k_{max} (T_o)] = 0.025 (0.201) (32.2) (0.7) = .113 feet

THEREFORE, FOR THE EXAMPLE PROBLEM THE AMOUNT OF DEFORMATION WOULD BE 0.113 FEET WHICH WOULD BE CONSIDERED ACCEPTABLE FOR LEACHATE COLLECTION SYSTEM WITH COLLECTION PIPES.

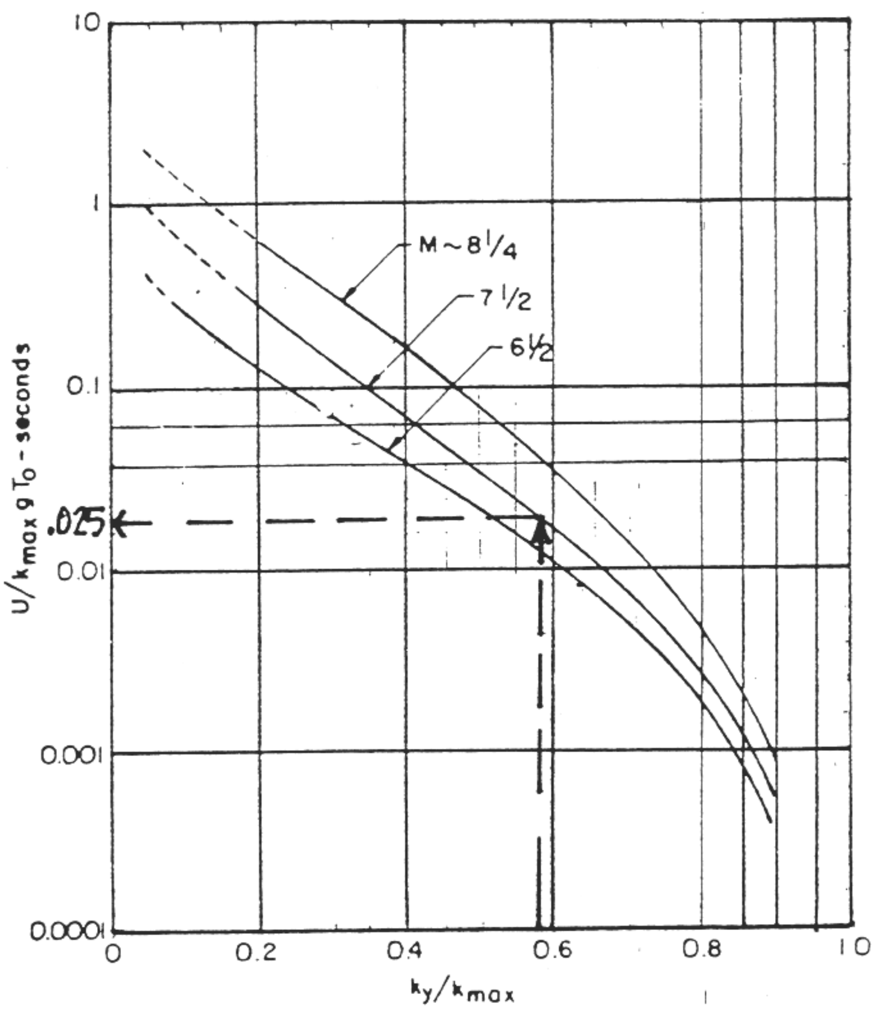


FIGURE 4: VARIATION OF AVERAGE NORMALIZED DISPLACEMENT WITH YIELD ACCELERATION

DETERMINATION OF LIQUEFACTION POTENTIAL

AND

THE IMPACT FOR LIQUEFACTION ON WASTE FILLS

DETERMINATION OF LIQUEFACTION POTENTIAL AND ITS IMPACT ON WASTE FILLS

The term liquefaction is used to describe a phenomenon in which COHESIONLESS soils liquefy so that the material takes on the characteristics of a fluid. The following procedure has been developed empirically to determine liquefaction potential and the potential impact on waste fills.

PROCEDURES FOR DETERMINATION OF LIQUEFACTION POTENTIAL AND ITS IMPACT ON WASTE FILLS

STEP 1. Determine the maximum horizontal acceleration (a_{max}) in g's from the USGS map number 2120 or from a site-specific seismic risk assessment.

STEP 2. Determine the total overburden pressure (P_{tot}) on the soil layer in question. (See Example Problem Two)

STEP 3. Determine the effective overburden pressure (P_o) on the soil layer in question. (See Example Problem Two)

STEP 4. Use Figure 5 to correct the standard penetration resistance value (N) for the effect of overburden pressure.

$$N_1 = C_N * N$$

(C_N is a correction factor based on the effective overburden stress.)

STEP 5. Determine the stress reduction factor (r_d) from Figure 6.

STEP 6. Compute the cyclic stress ratio, (R(), developed in the field during design earthquake:

$$R(= \tau_{av} / P_o = 0.65\, (a_{max})\, (P_{tot} / P_o)\, r_d$$

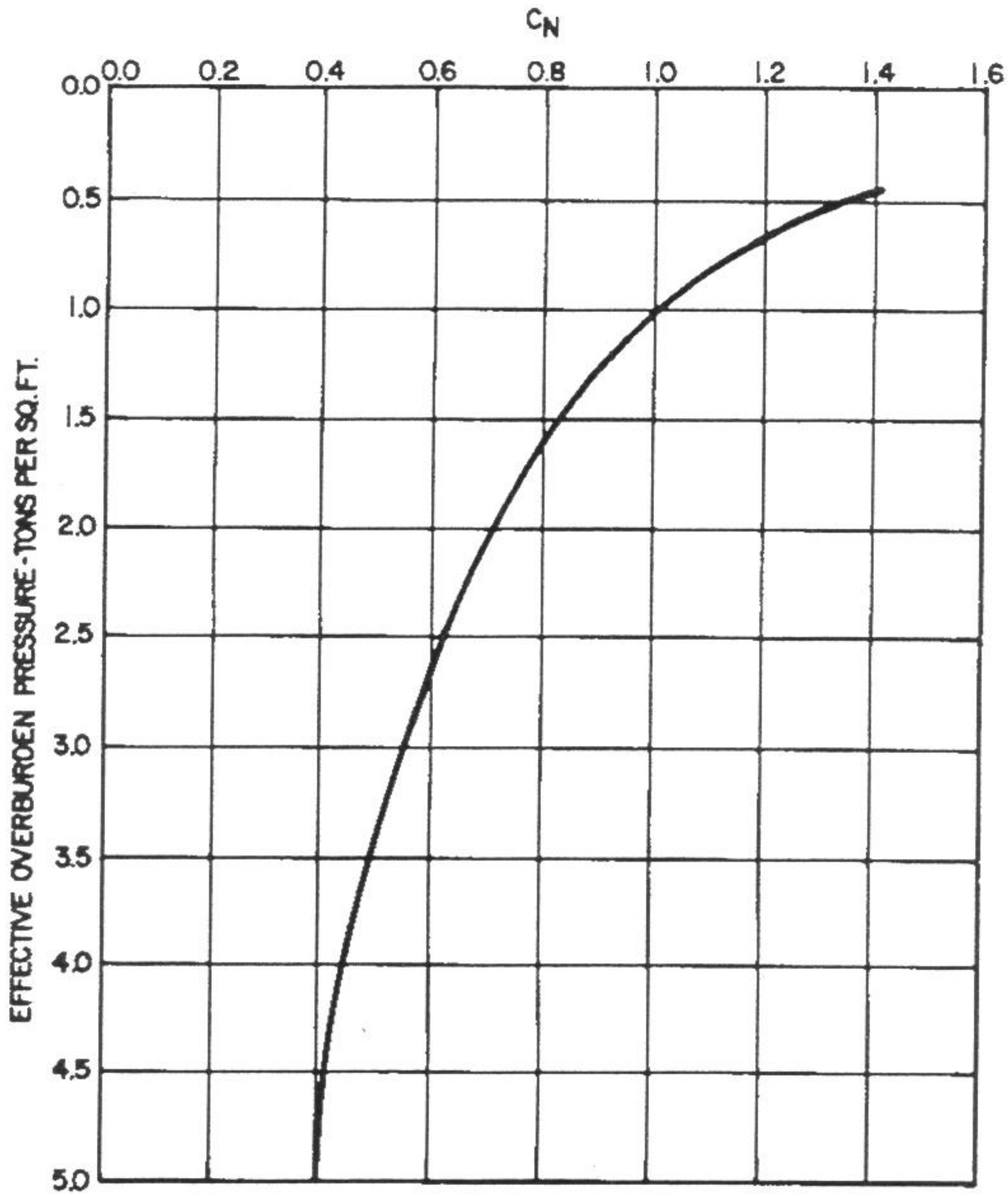
STEP 7. Knowing the magnitude of the earthquake (M) and (N_1) estimate the cyclic stress ratio R_f required to cause liquefaction from Figure 7.

STEP 8. Calculate the factor of safety against liquefaction F_s for each layer, to obtain an appropriate factor of safety which for earthen structures is generally between 1.2 and 1.5.

$$F_s = R_f / R_i$$

PROCEDURE FOR DETERMINATION OF LIQUEFACTION POTENTIAL AND ITS IMPACT ON WASTE FILLS (CONTINUED)

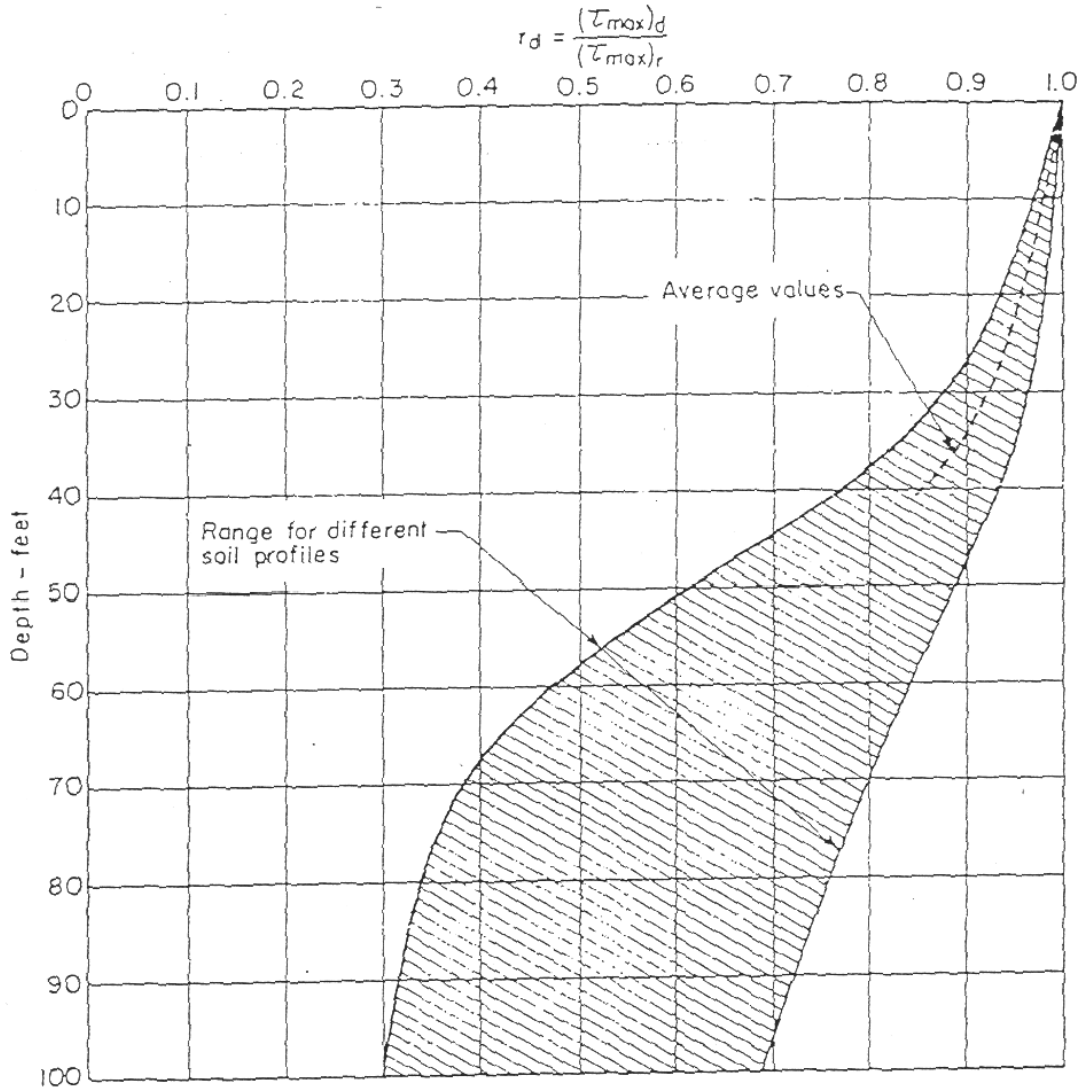
STEP 9. If the preceding steps do indicate that there is a potential for liquefaction within a particular layer it will be necessary to determine if the liquefaction will result in damage to the solid waste facility. The occurrence of liquefaction within a layer does not necessarily result in subsidence or deformation on the surface. In order to evaluate the potential for damage at a facility it will be necessary to enter Figure 8 and determine if the liquefaction will result in damaging movements on the surface.



Correlation Between C_N and Effective Overburden Pressure

FIGURE 5

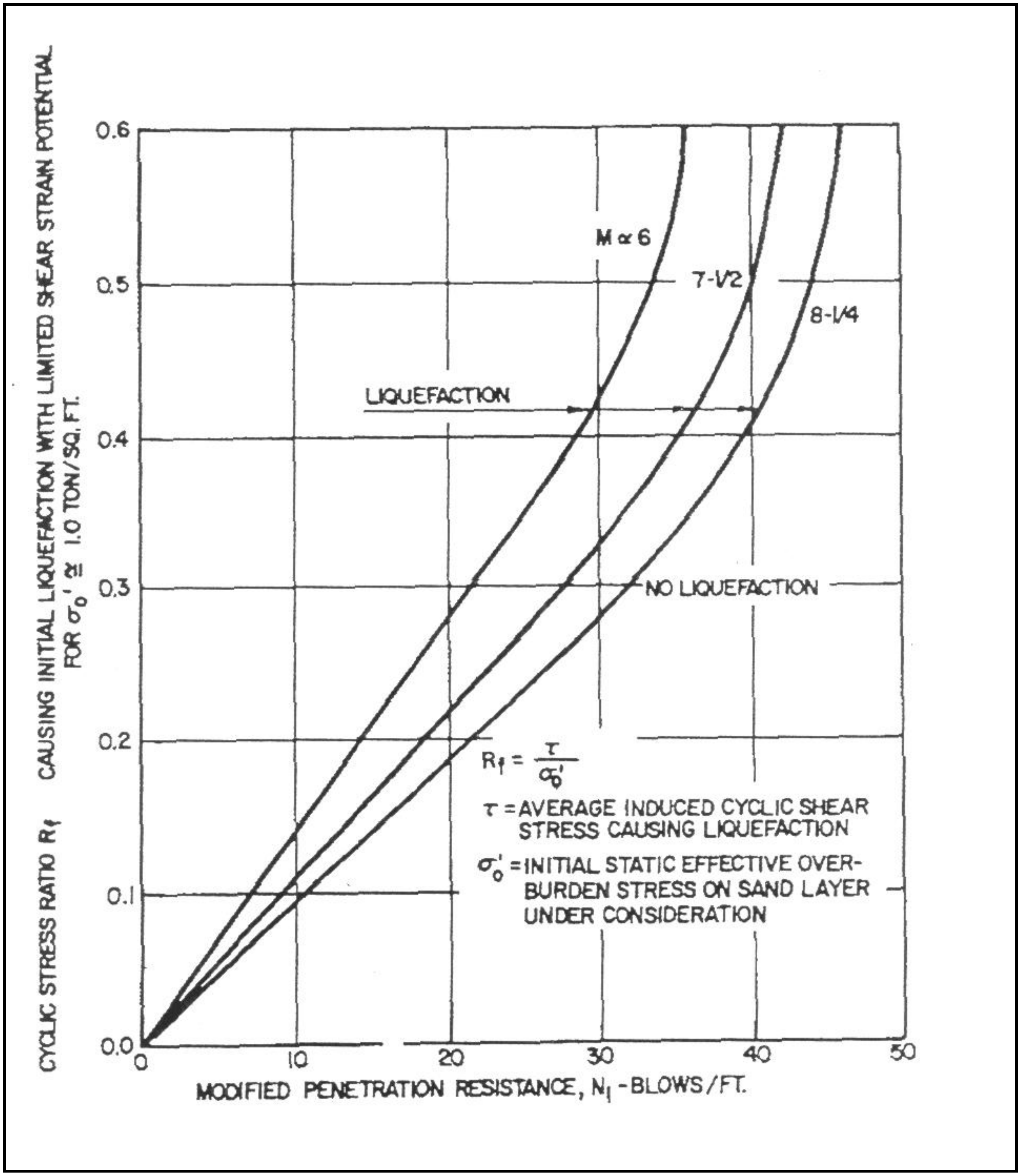
(FROM NAVFAC 7.3)



—RANGE OF VALUES OF r_d FOR DIFFERENT SOIL PROFILES

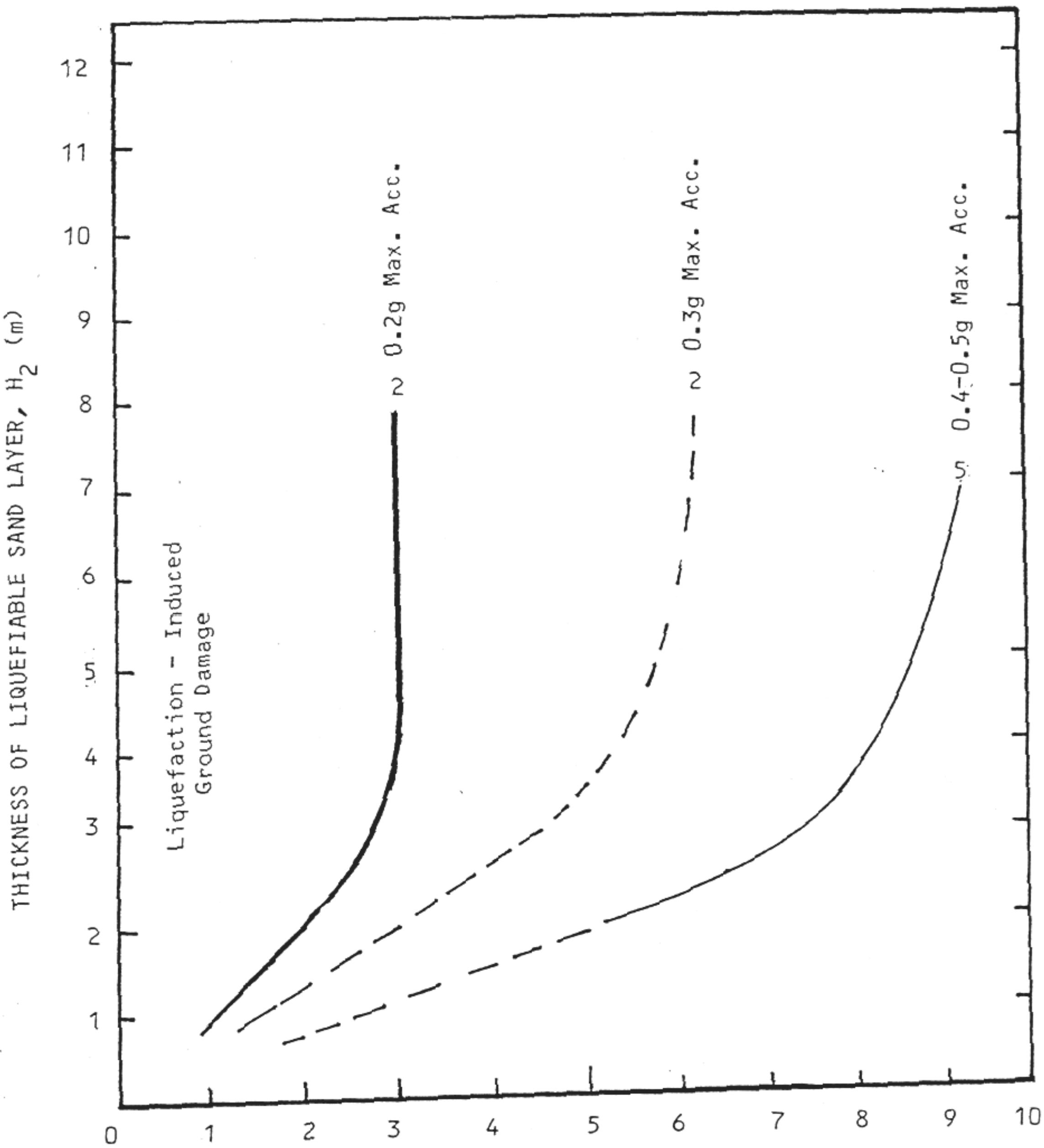
RANGE OF VALUES OF r_d FOR DIFFERENT SOIL PROFILES

FIGURE 6
(FROM H. B. SEED)



Correlation Between Field Liquefaction Behavior of Sands for Level Ground Conditions and Modified Penetration Resistance

FIGURE 7
(FROM NAVFAC 7.3)



THICKNESS OF SURFACE LAYER, H1 (m)

FIGURE 8
BOUNDARY CURVES FOR LIQUEFACTION SURFACE EVENT (Ishihara, 1985)

Maximum Surface Acceleration = 0.22g

APPENDIX II

CALCULATE OVERBURDEN PRESSURE

EXAMPLE PROBLEM TWO

KEY TO SYMBOLS

γ = unit weight of soil (lb. / cu. ft.)
γ_{wet} = wet unit weight of soil (lb. / cu. ft.)
γ_{sub} = submerged unit weight of soil (lb. / cu. ft.)
γ_{sub} = weight of soil below water table
$\gamma_{sub} = \gamma_{wet}$ **-** unit weight of water (62.4 lb. / cu. ft.)

P_{tot} = total overburden pressure
P_o = effective overburden pressure

Total overburden pressure (P_{tot}) is found by multiplying the wet unit weight of soil (γ_{wet}) by the thickness of each soil layer and continuously summing the results with depth.

Effective overburden pressure (P_o) is found as follows:

1. For soils above the water table multiply the wet unit weight of soil by the thickness of each respective soil layer above the desired depth. (Note: this is actually the total overburden pressure since it is above the water table.)

2. For soils below the water table reduce the wet unit weights by the weight of water (62.4 lb./cu. ft.) to determine the submerged unit weight (γ_{sub}). Then multiply the submerged unit weights by the thickness of each soil layer beneath the water table and the desired depth.

Example: Determine (P_o) at 20 feet below the ground surface in a silty clay deposit with a wet unit weight of 127 lbs./cu. ft. and the water table at 10 feet below the ground surface.

step one 10 ft. x 127 lb. / cu. ft. = 1270 lbs. / sq. ft. (P_o above water table)

step two 10 ft. x (127 - 62.4) = 646 lb./sq. ft. (P_o below water table)

step three Add the results for step one and step two

P_o = effective overburden pressure at 20 ft. = 1270 psf + 646 psf
P_o = 1916 psf

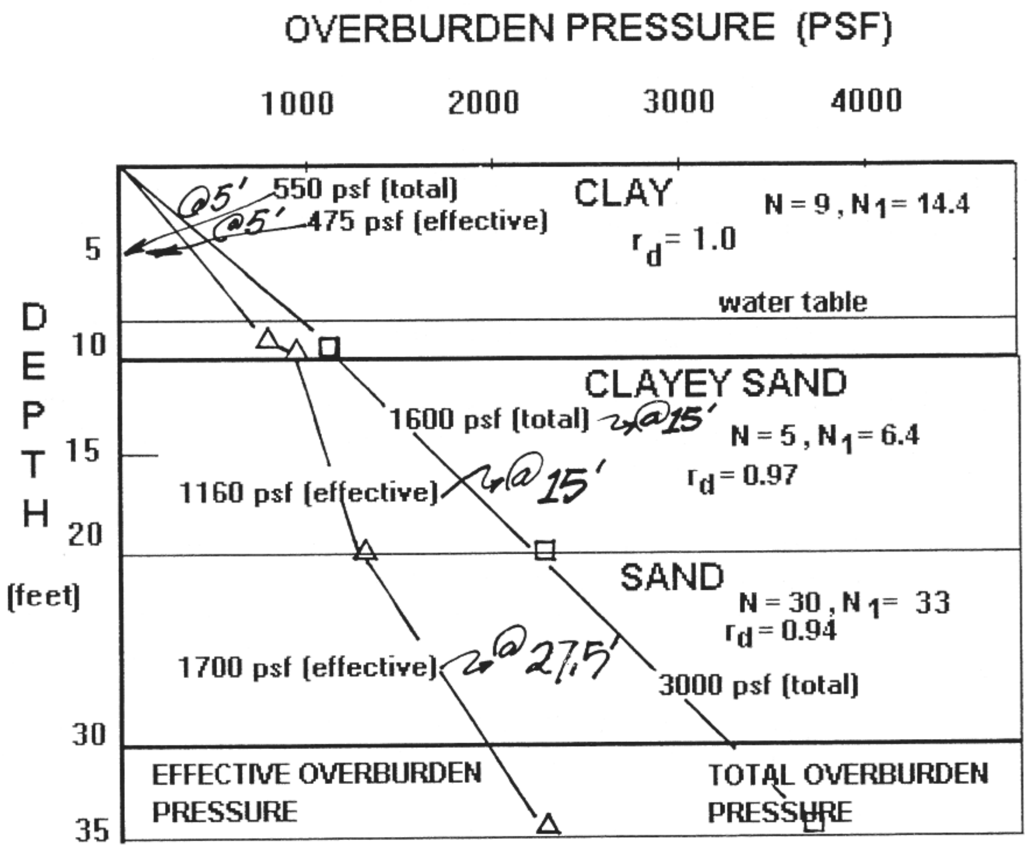
A plot of the effective overburden pressure versus depth is called a P_o diagram and is illustrated on the following page.

LIQUEFACTION EXAMPLE PROBLEM

EXAMPLE PROBLEM THREE

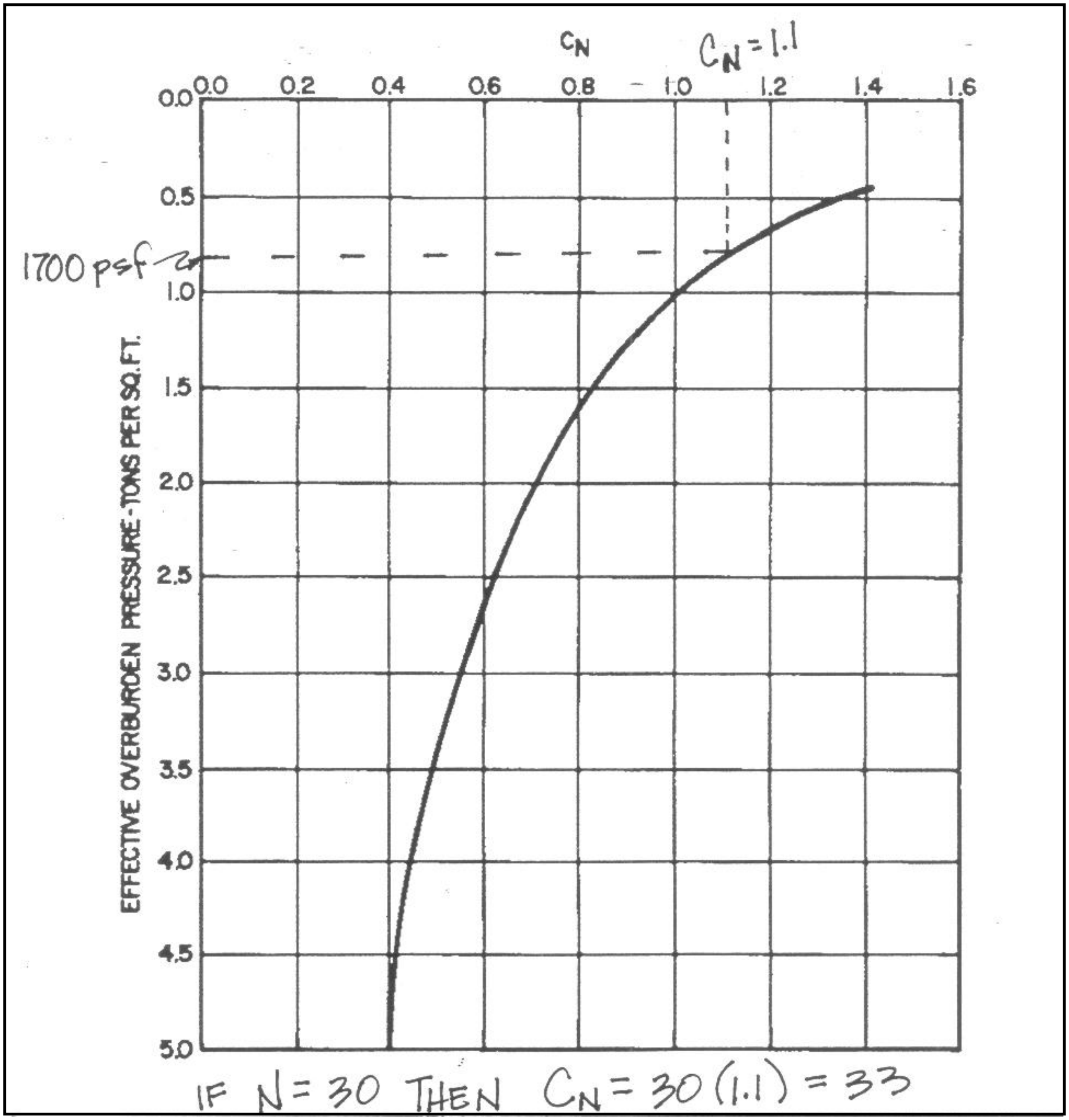
STEP 1. Determine the maximum horizontal acceleration (a_{max}) in g's from the USGS map number 2120

STEP 2. & 3. Determine the total overburden pressure (P_{tot}) and effective overburden pressure (P_o) on the soil layer in question.



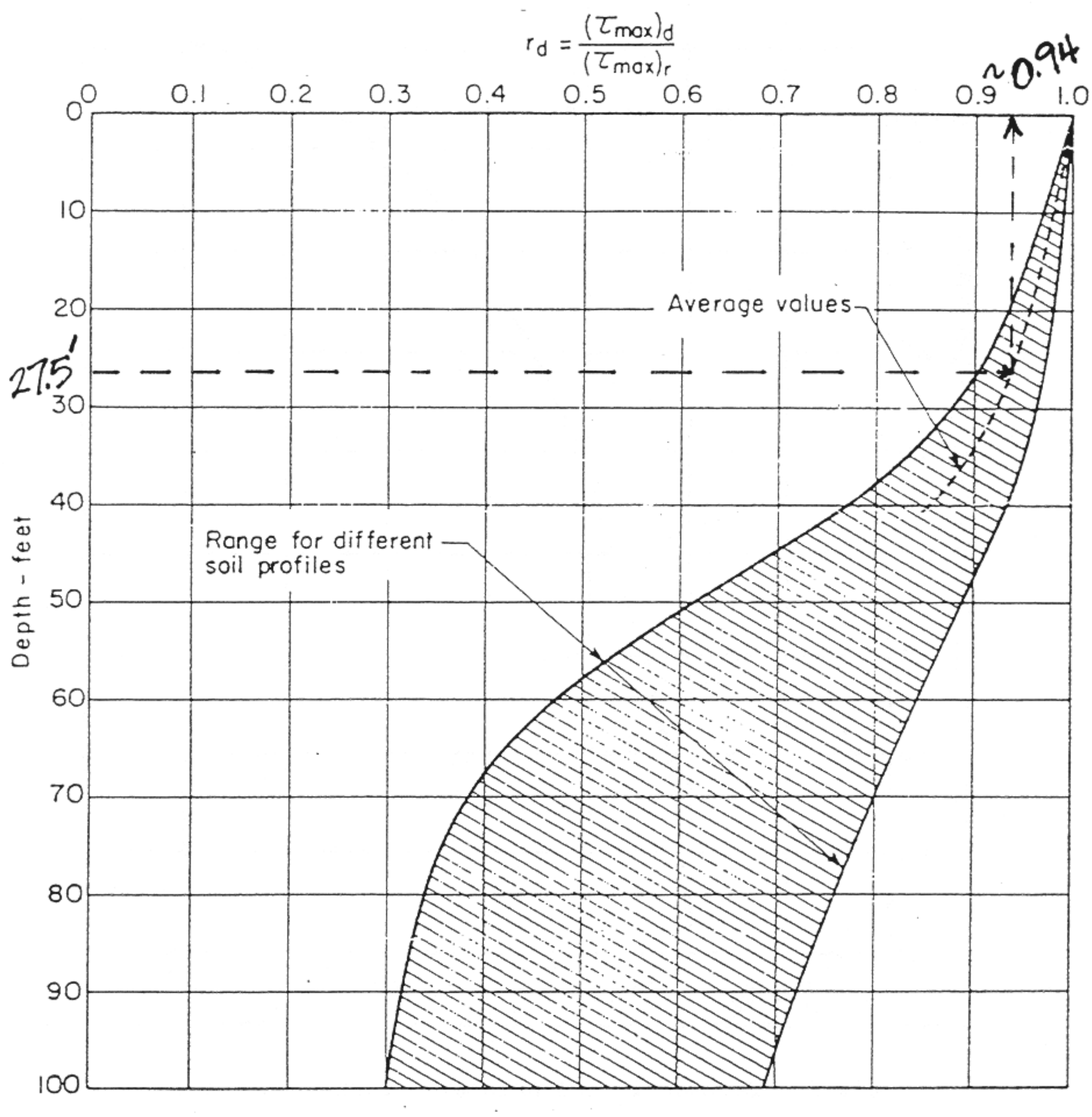
STEP 4. Correct N Values From Figure 5.

STEP 5. Determine Stress Reduction Factor (r_d) From Figure 6.



Correlation Between CN and Effective Overburden Pressure

FIGURE 5
(FROM NAVFAC 7.3)



RANGE OF VALUES OF r_d FOR DIFFERENT SOIL PROFILES

FIGURE 6

(H. B. SEED)

LIQUEFACTION EXAMPLE PROBLEM (CONTINUED)

STEP 6. Compute the Cyclic Stress Ratio (R_i).

$R(= \tau_{av} / P_o = 0.65 (a_{max}) (P_{tot} / P_o) r_d$

At 5 feet NOT APPLICABLE[1]

At 10 feet R(= 0.65 (0.23) (1100/975) 0.98 = **0.165**

At 15 feet R(= 0.65 (0.23) (1600/1160) 0.97 = **0.20**

At 27.5 feet R(= 0.65 (0.23) (3000/1700) 0.94 = **0.25**

STEP 7. Estimate the Cyclic Stress Ration (R_i) from Figure 7.

Assume Earthquake Magnitude of M = 7.5

At 5 feet R_f is not applicable to clay soils (see footnote 1)

At 10 feet N_1[2] = 12.2 then from Figure 7 R_f = **0.125**

At 15 feet N_1[2] = 6.4 then from Figure 7 R_f = **0.6**

At 27.5 feet N_1[2] = 33 then from Figure 7 R_f = **0.35**

STEP 8. Calculate the Factor of Safety (F_S) Against Liquefaction

At 5 feet R_f is not applicable to clayey soils

At 10 feet $F_S = R_f / R_i = 0.125 / 0.165 = 0.76$

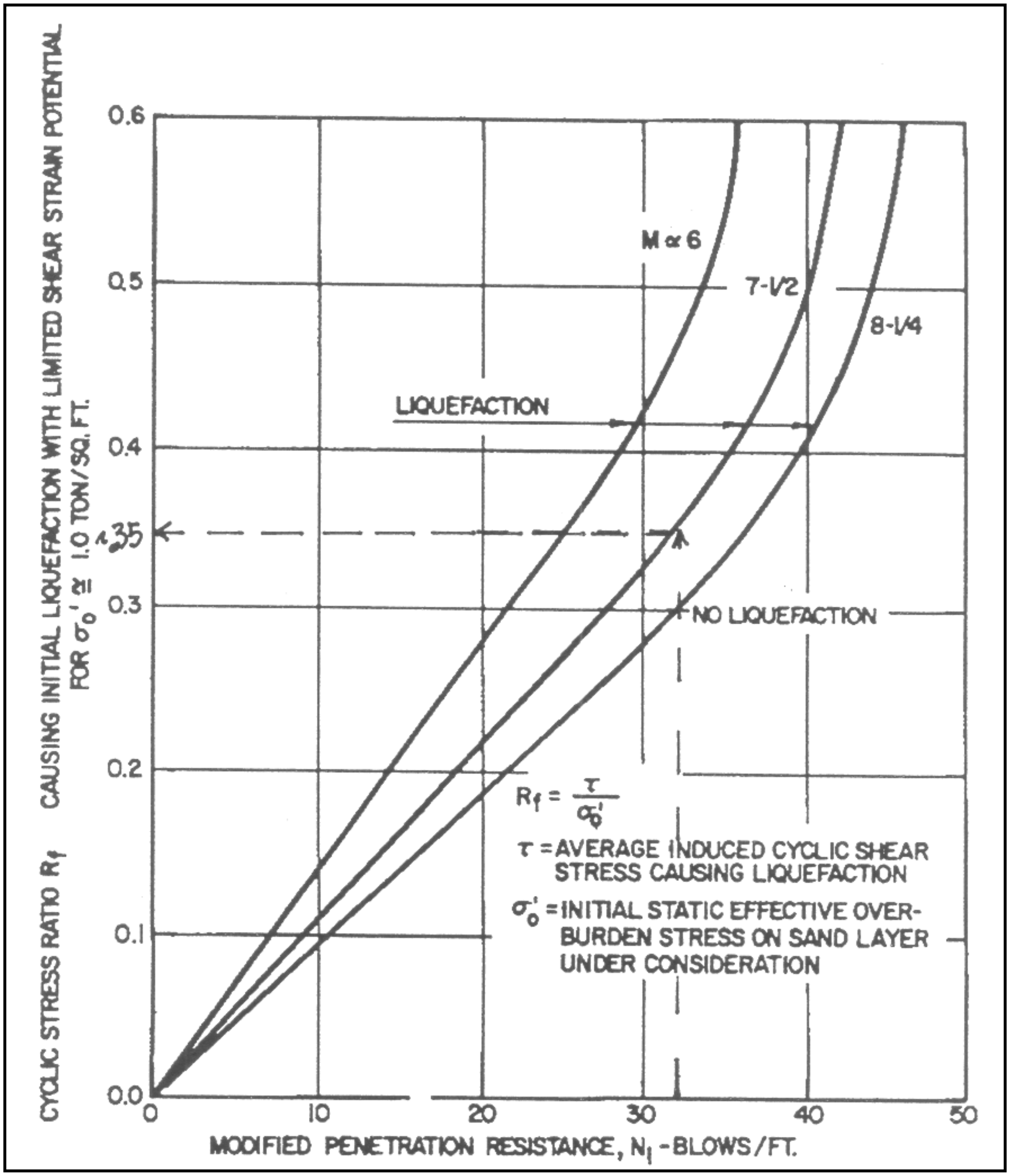
At 15 feet $F_S = R_f / R_i = 0.6 / 0.20 = 3.0$

At 27.5 feet $F_S = R_f / R_i = 0.35 / .25 = 1.4$

[1] **It has been shown that clay soils generally will not liquefy, however, recent studies conducted in China have shown that certain types of clayey soils are vulnerable to substantial strength loss as a result of earthquake vibration. These types of clayey soils generally have the following characteristics:**

Percent finer than 0.005 mm	**< 15%**
liquid limit	**<35**
Water Content	**>.9 liquid limit**

[2] N_1 represents the correct "N" value from the standard penetration test.



Correlation Between Field Liquefaction Behavior
of Sands for Ground Level Conditions and Modified
Penetration Resistance

FIGURE 7

(NAVFAC 7.3)

LIQUEFACTION EXAMPLE PROBLEM (CONTINUED)

STEP 9. Enter Figure 8 to determine if the predicted liquefaction at 10 feet will result in damaging surface movements.

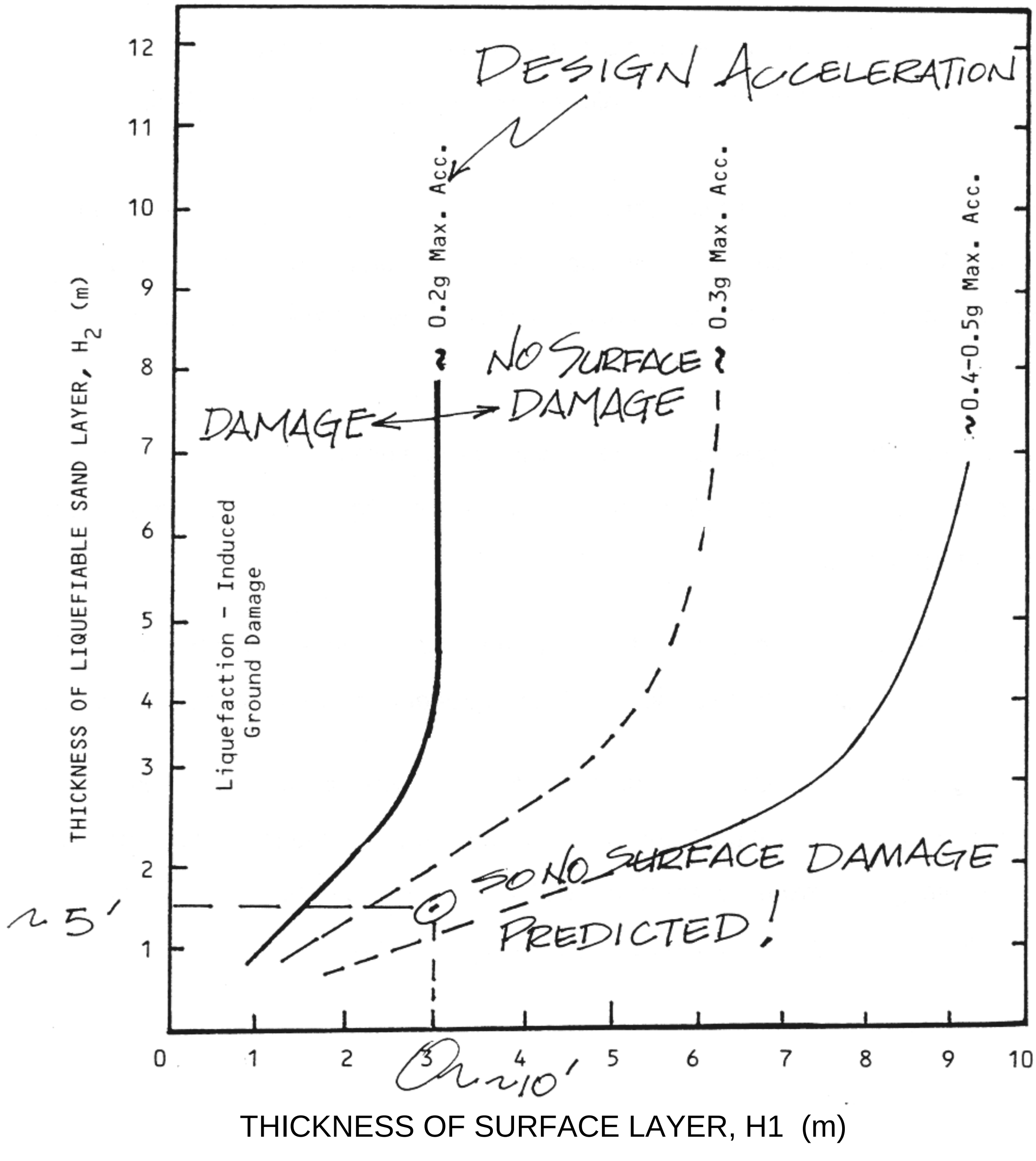


FIGURE 8

BOUNDARY CURVES FOR LIQUEFACTION SURFACE EVENT
(Ishihara 1985)

Maximum Surface Acceleration = 0.22g

SUMMARY OF EVALUATION POLICY

The state of the art relative to the effect that earthquake induced forces have upon solid waste landfills is still in its infancy. The procedure adopted in this policy is based upon empirical data and has been generally adopted by the industry. However, the Division is committed to stay current on any new findings relevant to earthquake analysis and will be open to new ideas.

In summary, the following limiting design criteria have been adopted by the Division of Solid Waste Management so as to insure that the landfill liner, leachate collection system and landfill appurtenances will remain functional when subjected to earthquake inducted forces.

I. Leachate collection systems and waste cells shall be designed to function without collection pipes for solid waste fill embankments that are predicted to undergo more than six inches of deformation.

II. No landfill shall be acceptable if the predicted seismic induced deformations within the waste fill exceed one-half the thickness of the clay liner component of the liner system.

III. In the event that liquefaction is predicted for a particular site it will be necessary to utilize Figure 8 to determine if the liquefaction will result in damaging movements on the surface. If damaging movements are predicted the site will be deemed unacceptable unless a plan can be implemented to densify the liquefiable layer.

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